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SYNOPSIS Larger scale dynamic centrifuge modeling has been used to examine the behavior of a soil-pile-structure system during earthquake-induced liquefaction. The model consisting of a 3 x 3-pile supported structure and a saturated sand deposit was constructed in a laminar container with the inside dimensions of 74-cm length, 50-cm height and 34-cm width. Test results show that liquefaction occurred only within a finite zone in the saturated sand deposit subjected to a strong input shaking that corresponds to a maximum earthquake acceleration of 0.3 g induced probably in actual ground, which agrees well with the results of earthquake damage investigation; moreover, larger induced bending strains of foundation piles were concentrated near the interface between liquefied and non-liquefied soil layers. This concentration of bending strains may be attributed to a remarkable reduction in shear resistance of liquefied soil layer relative to that of non-liquefied soil layer.

INTRODUCTION

Pile foundations have been extensively applied as an inexpensive method in order to mitigate the earthquake-induced hazards of various structures that were built on poor alluvial or reclaimed ground deposited over a large depth range and especially on soft saturated sandy ground. Serious damages of pile foundations occurred during the past earthquakes have been investigated, which were attributed to soil liquefaction (Kawasumi, 1968 and Kawamura et al, 1991). Accordingly, a number of shaking table tests have been performed on various models of soil-pile-structure systems to understand the mechanism of liquefaction-induced damages of pile foundations (e.g., Tatsuoka et al, 1978; Sato and Shamoto, 1989 and Tokimasu et al, 1991). However, it is believed to be very difficult by using model tests at 1 g field to simulate the dynamic behaviors of actual pile-supported structures during soil liquefaction, and most of these experimental results are seldom able to be directly used in the practice because these tests were conducted under the confining pressures quite smaller than those in the practical ground, and could not reproduce faithfully true in-situ stress field. It was observed from these model tests with lower confining pressures that complete liquefaction could emerge in model ground over its entire depth (Abe et al, 1991), which was not consistent with the results of case investigations for liquefaction-induced damages (Shamoto et al, 1987).

Recently, considerable attention has been attracted on dynamic centrifuge model tests that can closely reproduce the confining pressures in the practical ground. Several studies have been conducted for the purpose of examining the resistance of pile foundations to liquefaction during earthquake shaking, such as those by Miyamoto et al (1992), Koseki et al (1994) and Yoshizaki et al (1994). However, these tests appear to be not enough to reproduce quantitatively the dynamic behavior of a prototype soil-pile-structure system during liquefaction because there exist obvious lacks in the treatments for inertia effect and boundary effect when subjected to shear application with higher vibration frequency, the similitude requirements for model material permeability and model structures, the precision of simulating the input seismic waves and so on.

Presented in this paper are results from liquefaction tests of a model soil-pile-structure system. The tests were conducted under the conditions of satisfying similitude requirements of confining pressure and material permeability, by using a shaking table mounted on a dynamic geotechnical centrifuge that can provide larger payload and model space and closely reproduce both boundary constraint conditions of prototype ground and acceleration time histories of actual earthquakes. Based on the results from the tests, it has been found that complete liquefaction happened only within a finite zone in saturated model sand ground when subjected to a large scaled input earthquake shaking that corresponds to a maximum earthquake acceleration of 0.3 g induced probably in actual supporting deposits;

moreover, larger induced bending strains of foundation piles were concentrated near the interface between the liquefied and non-liquefied soil layers, which agree well with the results of damage investigation by exvacations after the past earthquakes.

TEST EQUIPMENTS AND MAIN FEATURES

Centrifuge Apparatus and Shaking Table

Model tests were conducted using the dynamic geotechnical centrifuge at the Institute of Technology, Shimizu Corporation, Tokyo. Fig.1 shows the plan and side view of the centrifuge. The arm radius of the centrifuge to the basket platform is 3.35 m for static tests and 3.11 m for dynamic tests. The centrifuge accommodates a maximum payload of 750 kg at the maximum acceleration of 100 g for static tests and 300 kg at 50 g for dynamic tests.

The centrifuge is equipped with a shaking table driven by an electro-magnetic shaker. The specifications of the shaking table tests are listed in Table 1. An electro-magnetic excitation system is adopted for shaking table tests on the centrifuge because it has an advantage over popular electro-hydraulic excitation system in that it is able to produce and control large acceleration over a wide frequency range in the same way as tests in 1 g field. Moreover, in order for the model to simulate prototype behavior during earthquakes, a computer-controlled digital feedback correction system is used to make the shaking table motion reproduce scaled earthquake time histories with satisfactory fidelity.

A Laminar Container

Special attention should be paid to the large inertia, probably too large displacement and boundary effect of a laminar container used in centrifuge tests, particularly in the case of liquefaction tests. Accordingly, it is necessary to make a laminar container as large as possible. Because the centrifuge used can provide larger space and payload capacity, a laminar container with the inside dimensions of 50-cm height, 34-cm width and 74-cm length (in shaking direction) was designed as shown in Fig.2. The container consists of 14 rectangular hollow frames made of square steel tubes. Linear bearings with a height of 2 mm were installed between these frames to reduce friction during shearing. The container was lined with 1 mm thick rubber membranes to provide waterproofing for saturated sand and to protect the bearings from the soil. The model ground can be saturated by supplying water or fluid with high viscosity through eight holes in the base plate.

Table 1 Specifications of shaking table test

Radius of rotation	3.11m
Size of shaking table	95cm×65cm
Centrifuge acceleration	5~50*g*
Maximum payload	300kg
Shaking acceleration	
: Sine wave	5*g*
: Random wave	10*g*
Range of frequency	50~350Hz

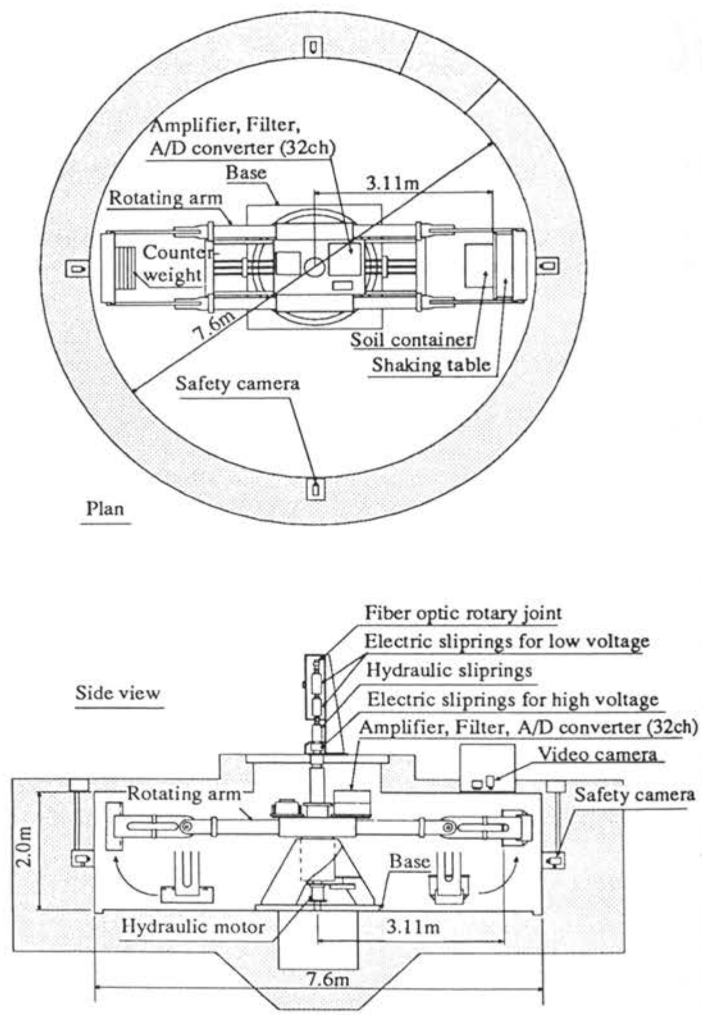


Fig. 1 Plan and side view of a geotechnical centrifuge

In addition to being rigid and strong, a container ought to be light as possible to eliminate its inertia effect. In the present design using 1.2 mm thick hollow steel tubes, the ratio between the mass of the container and that of the model ground is about 1:6 for saturated sand. The ratio is better than those adopted in the previous designs for centrifuge tests, although it is not as good as the usual ratio of about 1:20 for shaking table tests in 1 g field. It has been confirmed by the preparatory tests that the container could behave with good performance during cyclic shearing induced by input shakings; moreover, there was not significant inertia effect (Sato, 1994).

TEST CONDITIONS AND PROCEDURES

In this study, a soil-pile-foundation system was modeled to simulate such a case that a liquefiable saturated sand stratum was underlain by a bearing stratum, and a structure was built on such ground deposits and supported by group piles driven in the bearing stratum. Fig.2 shows the plan and side view of the structure-pile-foundation system constructed in the container. Shaking table tests on the model were conducted under 25 g. A scale of model to prototype was adopted as 1 : 25. The similitude requirements used in the tests are listed in Table 2. The specific test conditions and procedures will be described as follows.

Preparation of Model Foundation

The gravel, an uniformly graded material (D_{50} = 2.17 mm), was poured at a relative density of nearly 100%, to form an underlying bearing layer of the model ground. Subsequently, the dry Toyoura sand, of which the grading curve and physical properties are shown in Fig.3, was pluviated through a funnel from a constant height to control a relative density of 49%, finally forming a uniform overlying sand layer. As indicated in Fig.3, the model soil deposit was

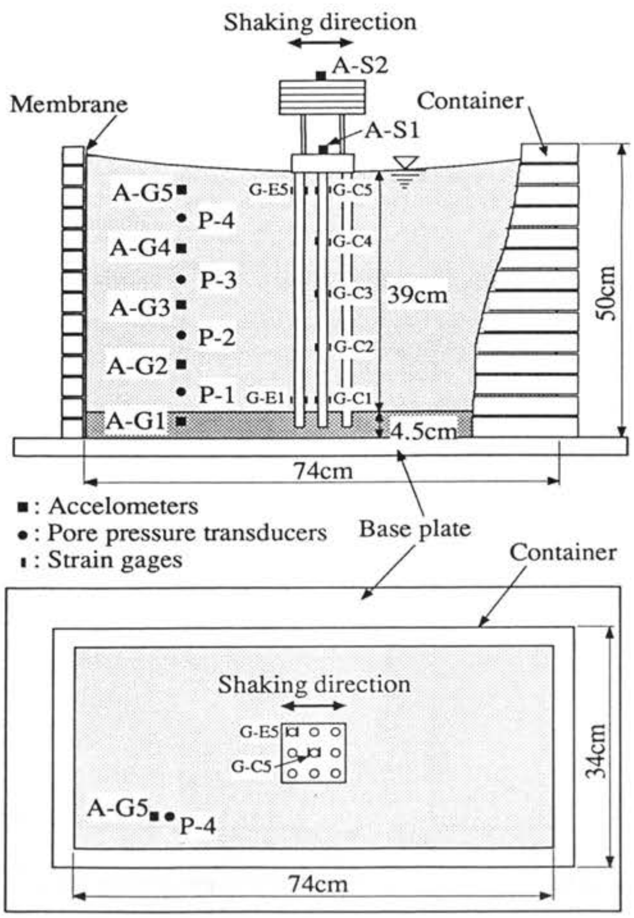


Fig. 2 A model of soil-pile-structure system prepared in a laminar container

Table 2 The similitude requirements used in the test

		Symbol	Scale ratio	Unit	Model	Prototype
Sand stratum	Thickness	H_g	$1/\lambda$	m	0.39	9.75
	Density	ρ_t	1	g/cm³	1.91	1.91
	Permeability	k	$1/\lambda$	cm/s	0.0012	0.03
Bearing stratum	Thickness	H_b	$1/\lambda$	m	0.045	1.125
Pile	Length of pile	L	$1/\lambda$	m	0.42	10.5
	Diameter	D	$1/\lambda$	cm	1.5	37.5
	Thickness	t	$1/\lambda$	mm	1.0	25
	Young's modulus	E	1	MN/m²	72000	72000
	Geometrical moment of inertia	I	$1/\lambda^4$	cm⁴	0.1083	42305
	Bending stiffness	E·I	$1/\lambda^4$	MNm²	7.8×10^{-5}	30.5
Footing	Mass	m_f	$1/\lambda^3$	kg	0.595	9300
	Length	L_f	$1/\lambda$	m	0.1	2.5
Structure	Mass	m_s	$1/\lambda^3$	kg	12.35	193000
	Natural frequency	f_s	λ	Hz	32.0	1.3
	Damping ratio	h_s	1	%	6.0	6.0
Exciting acclaration		α	λ	g	7.5	0.3

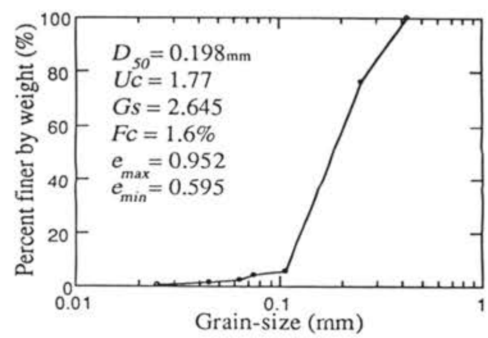


Fig. 3 Grain size accumulation curve and physical properties

constructed with a curved shape in order to simulate a prototype soil deposit with a flat surface.

A silicon oil with high viscosity, 30 times as viscous as water, was used instead of water to fill in the voids of model soils for the purpose of satisfying the similitude requirements. The prepared model, together with the laminar container, was input into a large vacuum box, and then, after the de-aired state in the box was formed, the liquid was allowed to seep upwards from the base plate at a very slow rate that required about 20 hours while under keeping vacuum. The moisture density and saturation degree of the model soil were 1.91 t/m³ and 99.3% respectively.

Modeling of Structure and Pile Foundation

The model structure was idealized by a rectangular layered steel plate with a mass of 12.35 kg and was supported with four platy springs. The springs were connected with a footing piece. Asphalt gum was positioned between the structure and the footing piece to provide a dashpot parallel to the springs. Such, the model structure could behave as a mass-spring-dashpot system sensitive to shear application. Natural frequency and damping ratio of the model structure were 32 Hz and 6%, which were measured by the shaking table test of the model structure under the condition that the bottom ends of four springs were fixed on the table.

Piles were made of aluminum pipe, 15 mm in diameter and 1 mm in thickness, and connected rigidly with the footing piece, constituting a 3 x 3 model group piles. Pile spacing was 2.5 times of pile diameter. The footing piece was a 0.595 kg platy hollow aluminum box (100 mm x 100 mm x 30 mm) within which a jointing material was filled. All the pile toes were inserted into the bearing stratum of gravel to the depth of two times as large as the pile diameter.

Measurements for Dynamic Response

Transducers were placed in the different positions of the model as described in Fig.2 to monitor and record dynamic responses of the pile-supported structure, piles and surrounding soil deposit during the test. The positions of pore pressure transducers and accelerometers were away from the piles in order to eliminate their effects on dynamic response of the piles; moreover, to examine the liquefaction behavior of model soil deposit. Response accelerations of the structure and the footing piece, bending strain and axial strain of the piles were measured during the test.

Preparatory Shaking Test and Liquefaction Test

Preparatory shaking table test was performed under centrifuge acceleration of 25 g before shaking table liquefaction test in order to evaluate the vibration characteristics of the soil-pile-structure system over a small strain range. In the test a random wave with a small maximum acceleration of 0.72 g was applied.

Subsequently, shaking table test of the soil-pile-structure system was carried out at 25 g. The saturated model sand ground could be liquefied by applying a scaled earthquake acceleration time record to shaking table. In the present test an acceleration-time history obtained during the 1983 Nihonkai Chubu earthquake at the Akita harbor was selected as an input seismic wave after the seismic time was compressed on a scale of one-twenty fifth and the peak acceleration was adjusted proportionally to 7.5 g corresponding to the peak acceleration of 0.3 g for a prototype soil deposit.

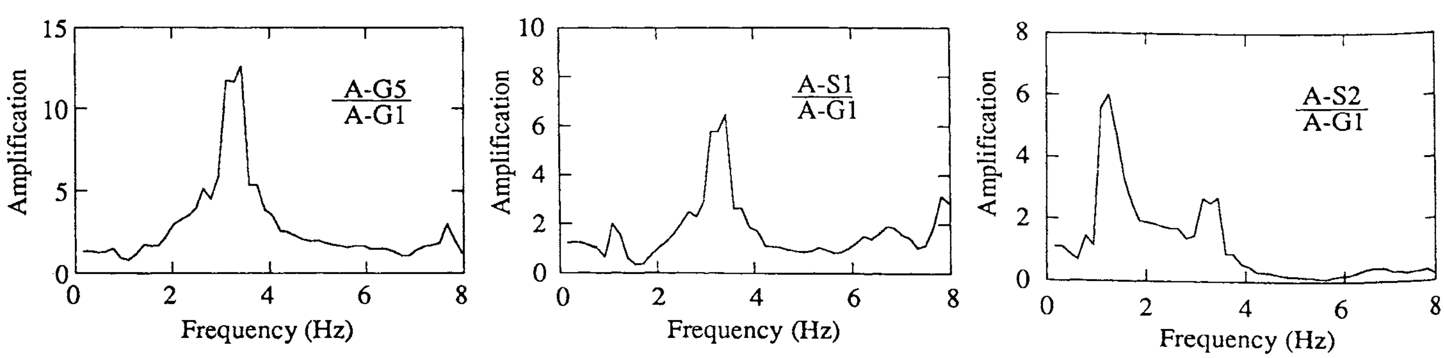
TEST RESULTS AND ANALYSIS

Vibration Characteristics of Soil-Pile-Structure System

Fig.4 shows the frequency transfer functions of the soil deposit near ground surface(at the point A-G5), the footing piece(A-S1) and the structure (A-S2) relative to the base plate (A-G1) respectively. In the figure the vibration frequencies indicate those of prototype soil-pile-structure system, of which the values were transformed from the data measured by the preparatory shaking table test, according to the similitude requirements listed in Table 2. From Fig.4 (a), (b) and (c), the first order of natural frequency of the soil deposit may be determined, which is about 3.2 Hz that corresponds to the peak response. The natural frequency of the structure is about 1.3 Hz as illustrated in Fig.4(c), less than that of the soil deposit. The experimental result shows that the model above could simulate the main features of dynamic response of a prototype structure-pile-soil system consisting of a pile-supported high-rise building and the surrounding soil deposit with a depth of about 10 meters.

Liquefaction Phenomenon in Sand Deposit

Time histories of response accelerations and excess pore pressures observed at the different positions in the sand deposit during shaking test are shown in Figs.5 and 6. According to Fig.6, the distribution of induced excess pore pressure ratio along the depth at the different moments during the shaking could be obtained as described in Fig.7. It can be seen from Fig.6 and Fig.7 that liquefaction occurred in the soil



(a) Ground surface (b) Footing (c) Structure

Fig. 4 Frequency response function at the three locations

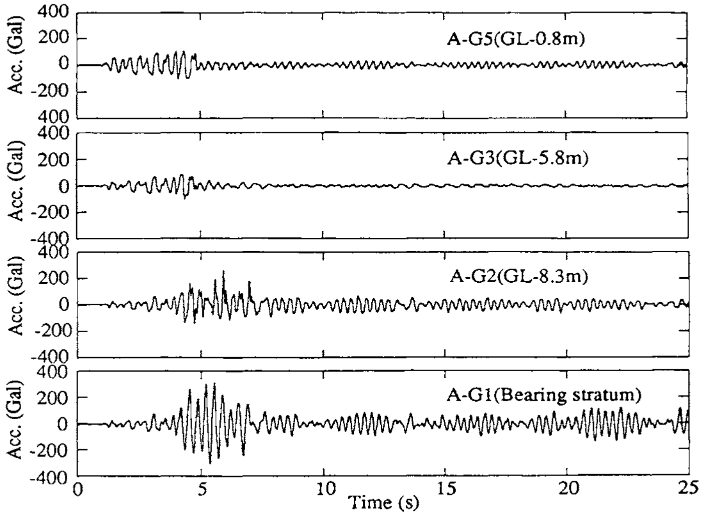


Fig. 5 Time history of measured response accelaration

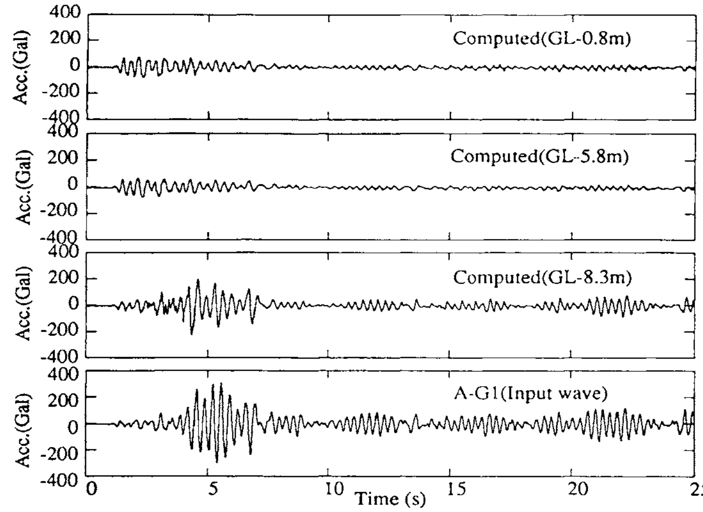


Fig. 8 Time history of computed response accelaration

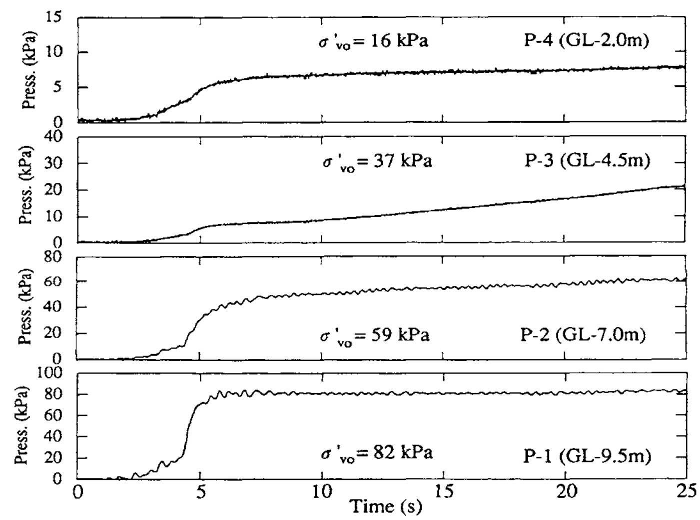


Fig.6 Time history of measured excess pore pressure

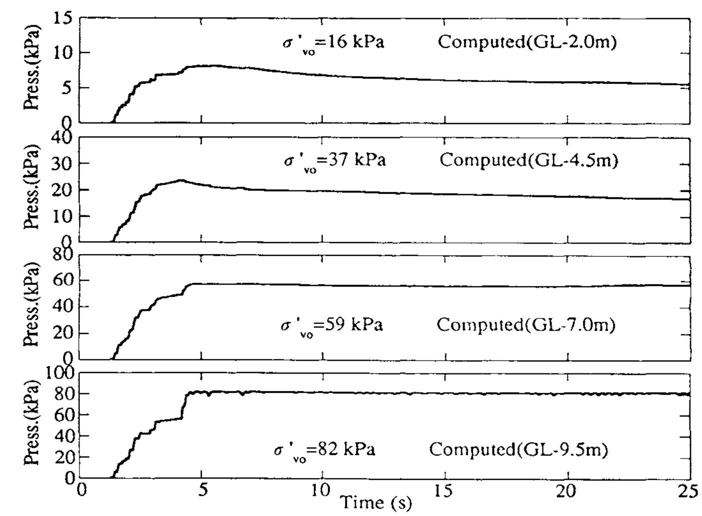


Fig.9 Time history of computed excess pore pressure

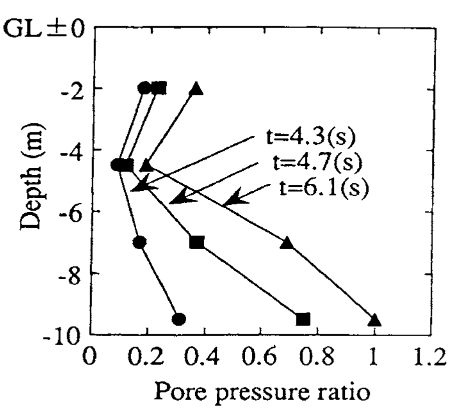


Fig.7 Distribution of excess pore pressure ratio along the depth

layer at the depth of 9.5 meters beneath the ground surface (written as the soil layer at G.L.-9.5 m) after about 6 seconds from the start of shaking, while induced ratios of excess pore pressure in the soil layers at G.L.- 7 m and G.L.-4.5 m rose to about 80% and 25%, respectively, at the time when liquefaction initially happened in the underlying soil layer at G.L.-9 m. After this time, although the intensity of input earthquake shaking remarkably decreased, the excess pore pressures in the two non-liquefied soil layers at G.L.-7 m and G.L.-4.5 m increased significantly and gradually because excess pore pressures diffused from the underlying liquefied soil layer across the soil layer of G.L.-7 m into the overlying soil layer at G.L.-4.5 m, while accompanied by the seepage flow of pore water from the bottom of soil deposit towards the ground surface. When the shaking time attained about 23 seconds, liquefaction took place in the soil layer at G.L.-7 m. This indicates that the test could reasonably reproduce the phenomenon of seepage-induced liquefaction that occurs in actual soil deposits.

By comparing Fig.5 and 6, it is found that response accelerations of the soil layer over a depth range from the ground surface to G.L.-5.8 m reduced to very small values after shaking 5 seconds; moreover, the response accelerations of longer period prevailed, which displayed a basic feature of acceleration response after liquefaction.

Figs. 8 and 9 shows the time histories of response accelerations and excess pore pressures in the soil deposit at the different depth which come from the results of one-dimensional dynamic response analysis of effective stress using the method proposed by Shamoto et al (1992). The parameters of soil property used in the dynamic response analysis were determined by using cyclic torsional shear test on hollow cylindrical specimens of Toyoura sand with a relative density of 50%. The acceleration-time history measured at the underlying bearing layer of gravel (A-G1) was adopted as an input earthquake wave for computation. By making a comparison between the measured and computed results shown in Figs.5, 6, 8 and 9, it was known that the measured and computed peak response accelerations and maximum ratios of excess pore pressure almost appeared at the same depth; moreover, liquefaction only occurred in the soil layer at G.L.-9 m. Such phenomenon that liquefaction happened only in a local zone of saturated deposits is consistent with the results of investigation for the liquefied soil deposits during the past earthquake (Shamoto et al, 1987).

Furthermore, in Fig.10 is shown the distribution of computed maximum shear strains along the depth of the soil deposit. It is known that larger shear strains appeared in the liquefied soil layer.

<u>Pile-Structure Response during Soil Liquefaction</u>

Fig.11 shows the acceleration time histories of the structure at the point A-S2 and the footing piece at the point A-S1 measured during shaking table liquefaction. Dynamic response of the footing was approximately same as that at the

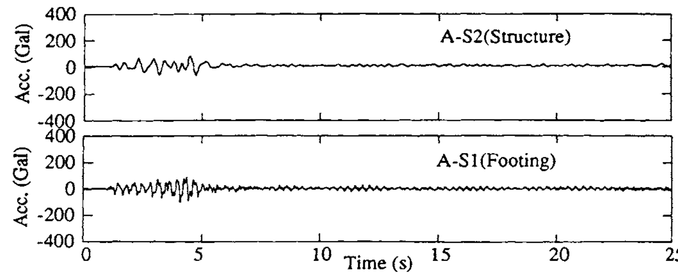


Fig. 11 Accleration-time histories of structure and footing

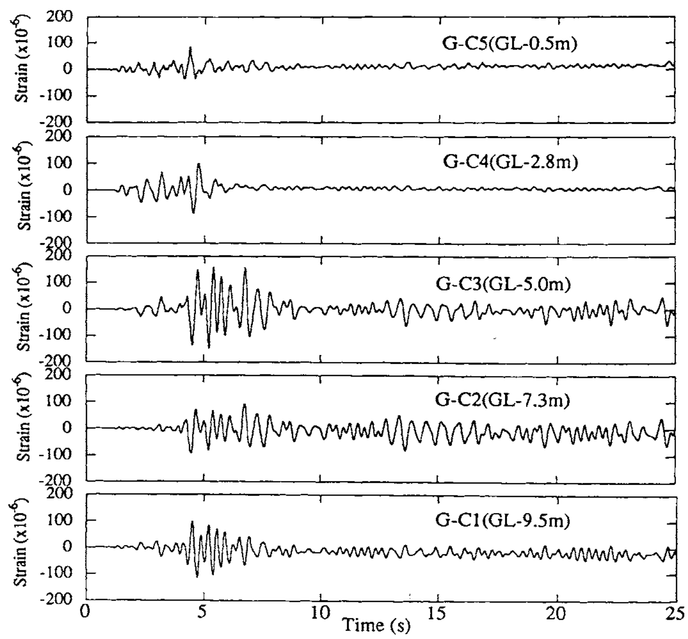


Fig. 12 Bending strain-time histories of piles

point A-G5 near the ground surface. Response acceleration of structure changed with the change in response acceleration of the footing because the response of the latter was actually regarded as an input to the former. Obviously, the response of the structure would approach to very small values after liquefaction appeared in the underlying soil layer. At this time, the response of

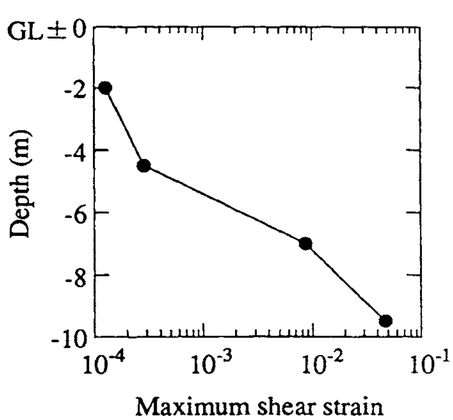


Fig. 10 The distribution of computed horizontal shear strain in the soil deposit

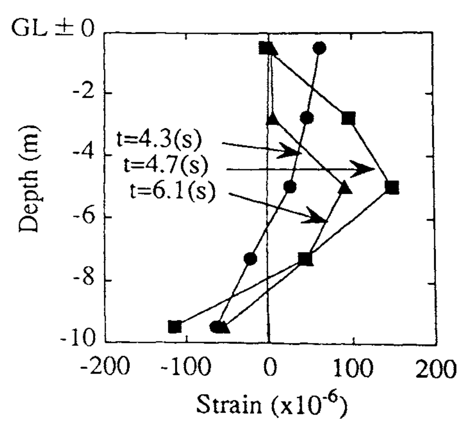


Fig. 13 The distribution of measured bending strain of pile along the depth

footing relative to an input that was transformed from the structure could be neglected, which is a significant conclusion for aseismic design of pile foundations.

In Fig.12 are shown bending strains of piles at the points from G-C1 to G-C5. The distributions of bending strains along the depth are illustrated in Fig.13. As can be found from the two figures, larger bending strains occurred near the interface between the non-liquefied and liquefied soil layers (at G.L.-5 m and G.L.-9.5 m). However, induced bending strains were relatively small in both the overlying non-liquefied soil layer (about G.L.-5 m) and the liquefied one (at G.L.-7.3 m). By comparing Fig.10 and Fig.13, it can be said that strains of piles were controlled by strains of soil deposits. Additionally, although the input acceleration decreased significantly after the seismic time attained about 10 seconds, bending strains of piles were still relatively large and could not be neglected because larger strains of the surrounding soil deposits occurred, which associated with the phenomenon that the response accelerations of long period prevailed in the liquefied soil layer. Hence, it is believed to be important that the evaluation for aseismic safety of pile foundations during soil liquefaction should be carried out not only when subjected to a larger input shaking, but also subjected to a relatively smaller shaking with a longer period after soil liquefaction.

Because the pile toes were inserted into the bearing layer of gravel, their displacements were restricted, which led piles to behave in a state between fixed and hinged supports during soil liquefaction. It is , therefore, believed that induced bending strains of piles near the interface between the sand layer and the gravel layer during the tests could reproduce those of actual foundation piles to a certain degree. Meanwhile, an interesting conclusion may be obtained: bending strains of piles were not effected significantly by the effect of inertia forces of the structure when liquefaction occurred in a finite zone of soil deposit over a certain depth beneath ground surface. The reason is probably because the input shaking was isolated by the liquefied soil layer so that the inertia effect of the structure obviously reduced. As a result as shown in Fig.11, the input that was transformed from the structure to the pile head could be neglected.

The above phenomenon that bending strains of piles were concentrated near the interface between the liquefied and non-liquefied soil layers could be explained by a remarkable reduction in shear resistance of the liquefied soil layer relative to the non-liquefied one. Such a remarkable reduction in shear resistance of the liquefied soil layer could result in an obvious difference in shear resistance near the interface between the liquefied and non-liquefied soil layers.

Fig.14 and Fig.15 shows measured time histories of bending and axial strains of the pile near its head and toe. Bending strains of pile near both its head and toe almost changed like those of the pile near its medium shown in Fig.12, because strains of piles were governed by strains of surrounding soil deposit. Moreover, larger axial strains of pile near its head and toe were nearly same phase because of locking induced by vibration of the structure.

Through comparing among Figs.12, 13, 14 and 15, it was known that the magnitudes of maximum bending and axial strains of piles were in the orders of 2×10^{-4} and 1×10^{-4} respectively, which were much smaller than the strain leading group piles to fail, although a larger shaking with a maximum acceleration of about 300 gal was

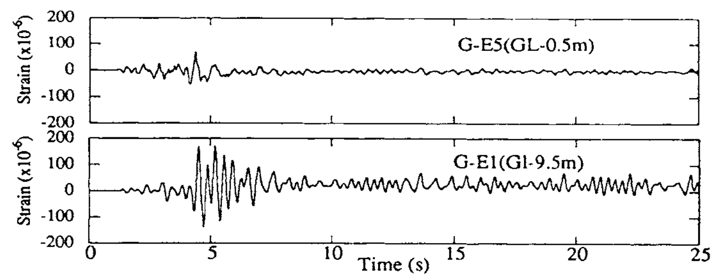


Fig. 14 Bending strain-time histories of pile near its head and toe

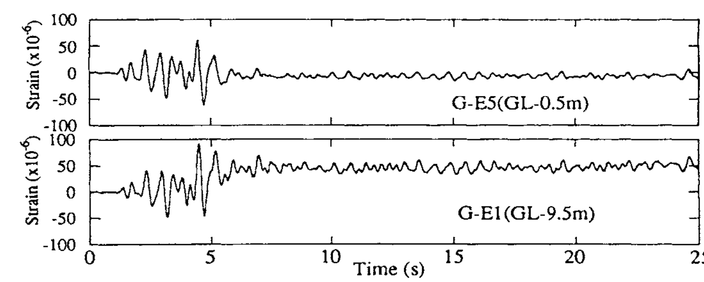


Fig. 15 Axial strain-time histories of pile near its head and toe

input near the bottom ends of piles. This indicates that for the case of a homogeneous ground with a level surface, there would almost not exist such a possibly that group piles were damaged during soil liquefaction if bearing piles had sufficient shear resistance, such as steel pipe pile was applied. Moreover, no significant settlement and inclination of the structure were observed after the liquefaction test.

CONCLUSIONS

The following conclusions may be obtained on the basis of a larger scale dynamic centrifuge test on a model of soil-pile-structure system constructed in a laminar container:

1. The model of soil-pile-structure system adopted could reasonably reproduce the dynamic

feature of actual pile-supported structures during soil liquefaction;

2. Liquefaction occurred only within a finite zone in saturated sand deposit at a certain depth beneath the ground surface subjected to a strong input earthquake shaking with a maximum acceleration of 0.3 g, which agrees well with the results of earthquake damage investigation;

3. The evaluation for aseismic safety of pile foundation during soil liquefaction should be carried out not only when subjected to a larger input shaking, but also subjected to a relatively smaller shaking with longer period after soil liquefaction;

4. Bending strains of piles were concentrated near the interface between liquefied and non-liquefied soil layers during soil liquefaction. A remarkable reduction in shear resistance of the liquefied soil layer relative to the non-liquefied soil layer and the associating larger shear strains of liquefied soil layer were responsible for the concentration of bending strains of piles, but the inertia effect of structure might be neglected during soil liquefaction that occurred in underlying soil layer.

5. Strains of piles were controlled by strains of the surrounding soil layer. However, for the case of homogeneous ground with a level surface, bending strains of rigid group piles during soil liquefaction were not large enough to lead the piles to fail when subjected to a strong input earthquake shaking with a peak acceleration of 0.3 g.

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